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Shear Strength of High-Strength Concrete Walls and Deep Beams

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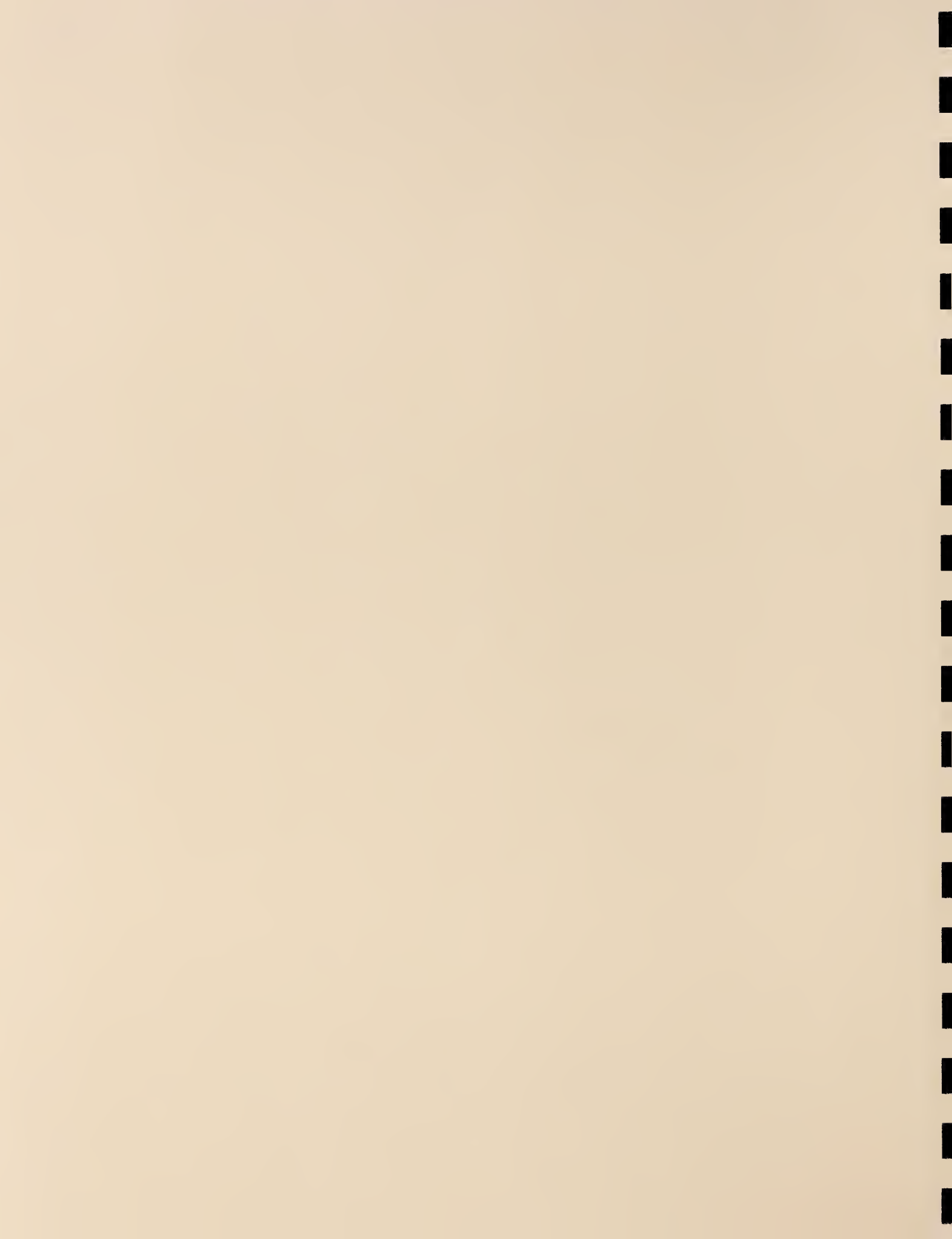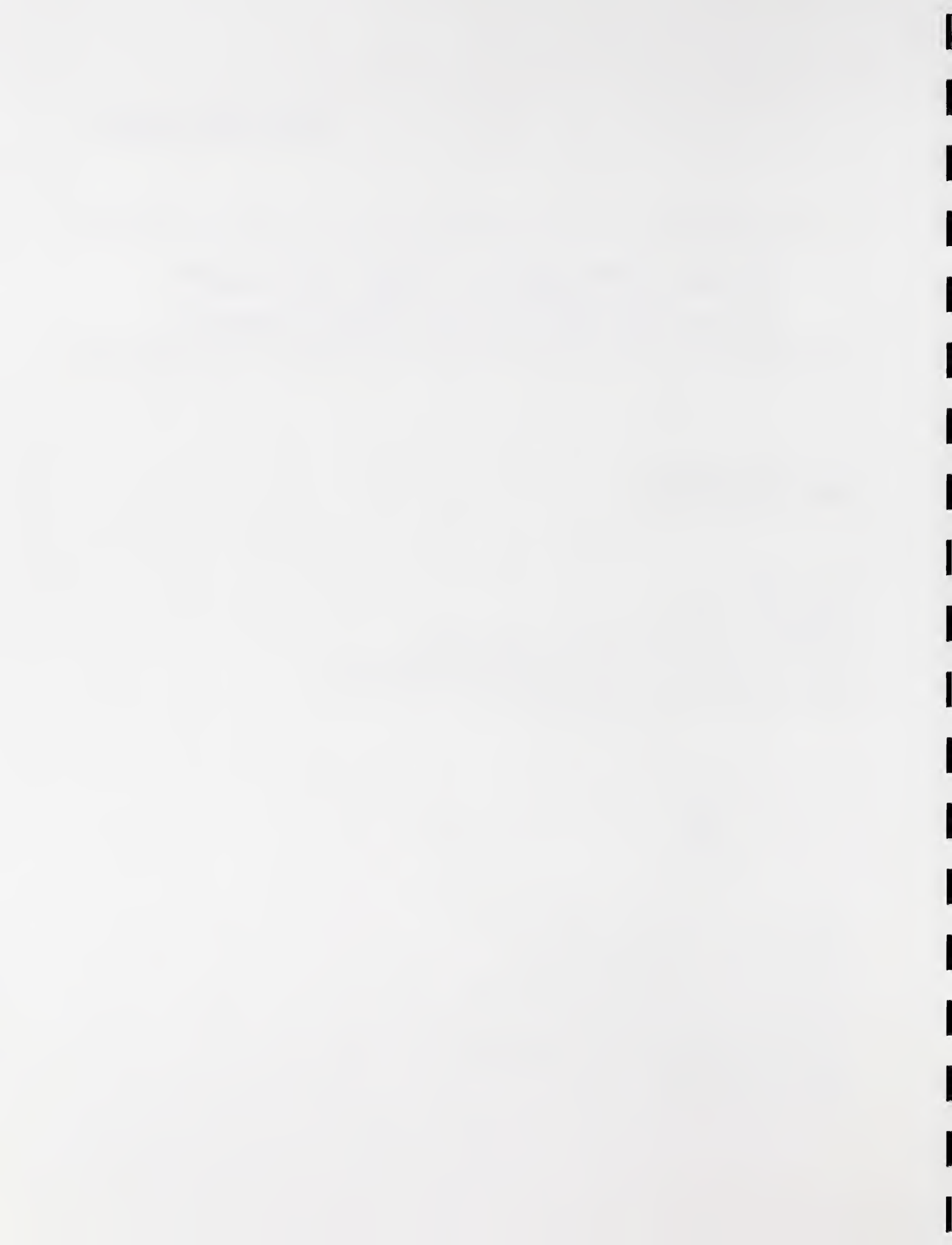Gaithersburg, Maryland 20899-8611

NIST

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ABSTRACT

In the last 30 years, the compressive strength of concrete that can be produced reliably in the field has more than doubled, from 35 MPa (5000 psi) to 85 MPa (12 000 psi). Strengths as high as 140 MPa (20 000 psi) can be achieved in the laboratory and, on rare occasions, in the field. Recently, the American Concrete Institute Subcommittee ACI 318-E (Standard Building Code, Shear and Torsion) set up a small working group, which included the author, to investigate barriers to the use of high-strength concrete related to shear strength provisions and to propose appropriate changes to the building code.

This report starts with a review of current shear strength provisions in ACI 318-99 for walls and deep beams. In light of recent tests, the author recommends lifting the ceiling of the square root of concrete compressive strength from the current value of 8.3 MPa (100 psi) to 11 MPa (130 psi) for code provisions that govern the shear strength of deep beams. However, there are insufficient data to propose any change to the current provisions governing the shear strength of walls.

Keywords: building code, cyclic tests, deep beams, high-strength concrete, shear strength, shear walls, static tests.
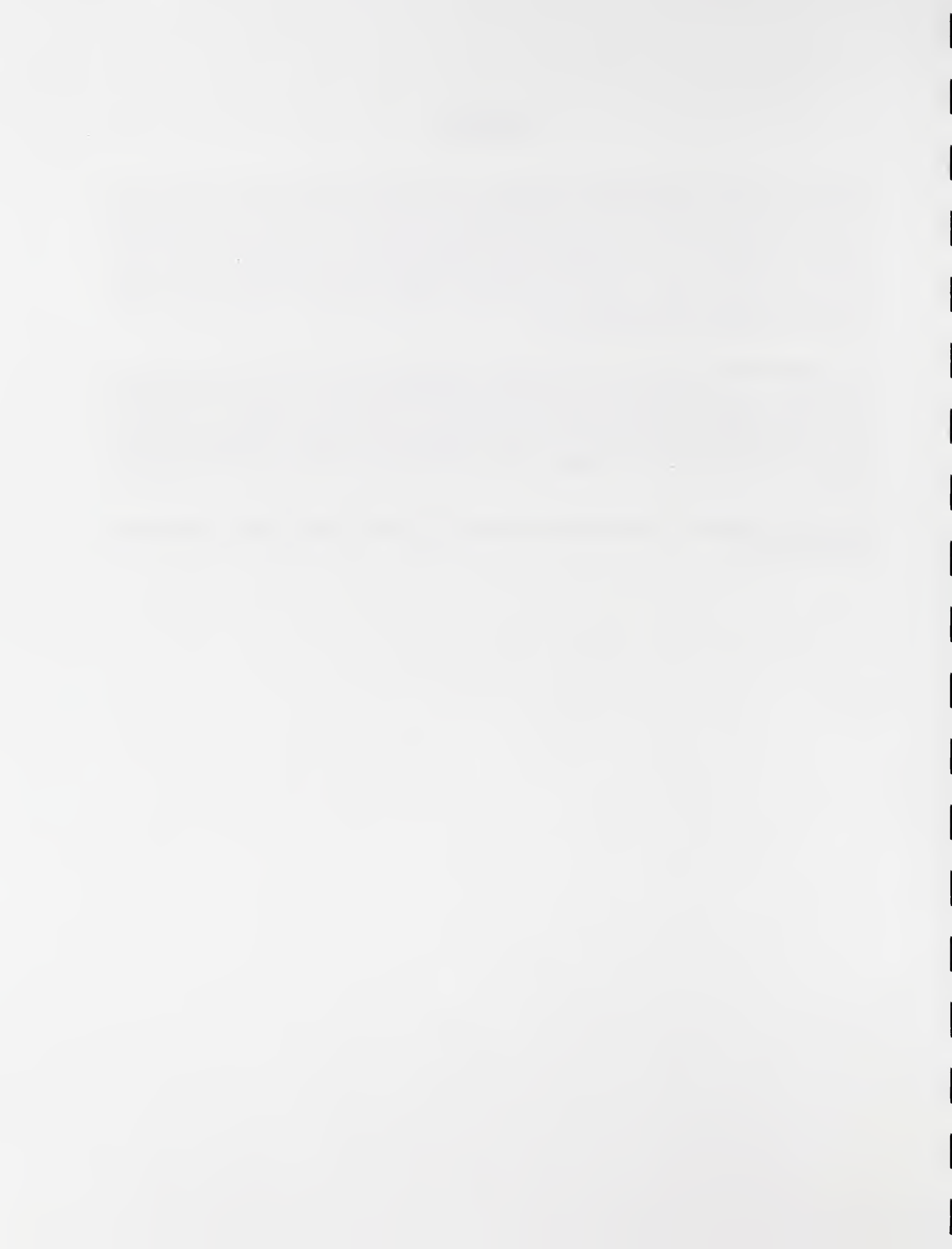
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1 INTRODUCTION

In the fall of 1998, the American Concrete Institute (ACI) Committee 318 (Standard Building Code), Subcommittee E (Shear and Torsion) formed a small task group to investigate barriers to the use of high-strength concrete in the shear design of structural members. The mission of the task group was to recommend to Committee 318 updates to Chapter 11, "Shear and Torsion", of ACI 318-99, in particular § 11.1.2, which limits the square root of the concrete compressive strength that may be used to calculate the concrete contribution to shear strength to 8.3 MPa (100 psi), unless minimum shear reinforcement is increased. The members of the task group consisted of Henry Russell (Henry G. Russell, Inc.), chairman, Dat Duthinh (NIST), Daniel Kuchma (University of Illinois, Urbana-Champaign), and David Sanders (University of Nevada, Reno). The purpose of this report is to document the task undertaken by NIST, namely, the update of ACI 318-99 shear strength design provisions to high-strength concrete (HSC) walls and deep beams.

2 DEEP BEAMS

2.1 Current Provisions

Current design provisions for shear strength of deep beams are presented in ACI 318-99 § 11.8 and apply to beams with l_n / d less than five, and that are loaded on one face and supported on the opposite face (§ 11.8.1. Notations are explained in Section 4). The shear strength of deep beams is limited by Eq. 11-27.

$$\frac{V_n}{\text{lbf}} \leq 8 \sqrt{\frac{f_c'}{\text{psi}}} \frac{b_w}{\text{in}} \frac{d}{\text{in}} \qquad \text{for} \qquad \frac{l_n}{d} < 2 \qquad (11\text{-}27a)$$

$$\text{or} \qquad \frac{V_n}{\text{N}} \leq \frac{2}{3} \sqrt{\frac{f_c'}{\text{MPa}}} \frac{b_w}{\text{mm}} \frac{d}{\text{mm}} \qquad \text{for} \qquad \frac{l_n}{d} < 2$$

$$\text{and} \qquad \frac{V_n}{\text{lbf}} \leq \frac{2}{3}\left(10 + \frac{l_n}{d}\right) \sqrt{\frac{f_c'}{\text{psi}}} \frac{b_w}{\text{in}} \frac{d}{\text{in}} \qquad \text{for} \qquad 2 \leq \frac{l_n}{d} \leq 5 \qquad (11\text{-}27b)$$

$$\text{or} \qquad \frac{V_n}{\text{N}} \leq \frac{1}{18}\left(10 + \frac{l_n}{d}\right) \sqrt{\frac{f_c'}{\text{MPa}}} \frac{b_w}{\text{mm}} \frac{d}{\text{mm}} \qquad \text{for} \qquad 2 \leq \frac{l_n}{d} \leq 5$$

The calculation of shear strength consists of the sum of the concrete contribution V_c and the steel contribution V_s, where:

$$\frac{V_c}{\text{lbf}} = \left(3.5 - 2.5 \frac{M_u}{V_u d}\right)\left(1.9 \sqrt{\frac{f_c'}{\text{psi}}} + 2500 \ \rho_w \frac{V_u d}{M_u}\right) \frac{b_w}{\text{in}} \frac{d}{\text{in}} \qquad (11\text{-}29)$$

or $\quad \dfrac{V_c}{N} = \left(3.5 - 2.5\dfrac{M_u}{V_u\,d}\right)\left(0.16\,\sqrt{\dfrac{f_c'}{\text{MPa}}} + 2500\ \rho_w\,\dfrac{V_u\,d}{M_u}\right)\dfrac{b_w}{\text{mm}}\,\dfrac{d}{\text{mm}}$

with $\quad \left(3.5 - 2.5\dfrac{M_u}{V_u\,d}\right) \le 2.5$

and $\quad \dfrac{V_c}{\text{lbf}} \le 6\,\sqrt{\dfrac{f_c'}{\text{psi}}}\,\dfrac{b_w}{\text{in}}\,\dfrac{d}{\text{in}}$

or $\quad \dfrac{V_c}{N} \le \dfrac{1}{2}\,\sqrt{\dfrac{f_c'}{\text{MPa}}}\,\dfrac{b_w}{\text{mm}}\,\dfrac{d}{\text{mm}}$

M_u is the factored moment occurring simultaneously with V_u at the critical section.
The steel contribution is:

$$V_s = \left[\dfrac{A_v}{s}\left(\dfrac{1+\dfrac{l_n}{d}}{12}\right) + \dfrac{A_{vh}}{s_2}\left(\dfrac{11-\dfrac{l_n}{d}}{12}\right)\right]f_y\,d \tag{11-30}$$

2.2 Tests of HSC Deep Beams
Tan et al. (1998) reviewed 233 deep beam tests classified as follows:

> 133 low-strength deep beams with $f_c' < 40$ MPa (5 800 psi),
> 32 medium-strength deep beams with 40 MPa (5 800 psi) $\le f_c' \le 55$ MPa (8 000 psi), and
> 68 high-strength deep beams with 55 MPa (8000 psi) $\le f_c' \le 120$ MPa (17 400 psi).

All beams had shear span to depth ratios $a/d < 2.5$, were loaded on top by one or two loads, and used normal weight aggregates. Results show that the shear strength provisions in ACI 318-99 produce conservative predictions (Table 1, Fig. 1).

Based on 16 tests of deep beams with 50 MPa (7 250 psi) $\le f_c' \le 120$ MPa (17 400 psi), Foster and Gilbert (1998) concluded that the shear strength provisions in ACI 318-99 were conservative (Figs. 1 and 2, Tables 2, 3, 4 and 5). Fig. 1 shows no correlation between the shear strength ratio and f_c'.

2.3 Recommendations
Based on the tests mentioned in the previous section, we recommend to raise the allowable concrete compressive strength for the shear strength of deep beams in ACI 318-99 § 11.1.2 to 120 MPa (17 400 psi). In terms of $\sqrt{f_c'}$, approximate $\sqrt{120} \approx 11$ and revise § 11.1.2 to read:

11.1.2 The values of $\sqrt{f_c'}$ used in this chapter shall not exceed 8.3 MPa (100 psi) except as allowed in 11.1.2.1 and 11.8.1.1.
Add a new provision as follows:

11.8.1.1 The values of $\sqrt{f_c'}$ used in 11.8 shall not exceed 11 MPa (130 psi).

3 Walls

3.1 Current provisions

Current provisions for the static shear strength of walls are found in ACI 318-99 § 11.10. The strength V_n for shear in the plane of the wall is limited to (§ 11.10.3):

$$\frac{V_n}{\text{lbf}} \leq 10 \sqrt{\frac{f_c'}{\text{psi}}} \frac{h}{\text{in}} \frac{d}{\text{in}}$$

or

$$\frac{V_n}{\text{N}} \leq 0.83 \sqrt{\frac{f_c'}{\text{MPa}}} \frac{h}{\text{mm}} \frac{d}{\text{mm}}$$

The shear strength V_n is the sum of the steel contribution V_s, and the concrete contribution V_c, which is the lesser of Eqs. 11-31 and 11-32:

$$\frac{V_c}{\text{lbf}} = 3.3 \sqrt{\frac{f_c'}{\text{psi}}} \frac{h}{\text{in}} \frac{d}{\text{in}} + \frac{N_u}{\text{lbf}} \frac{d}{4\, l_w} \tag{11-31}$$

or

$$\frac{V_c}{\text{N}} = 0.27 \sqrt{\frac{f_c'}{\text{MPa}}} \frac{h}{\text{mm}} \frac{d}{\text{mm}} + \frac{N_u}{\text{N}} \frac{d}{4\, l_w}$$

and

$$\frac{V_c}{\text{lbf}} = \left[0.6 \sqrt{\frac{f_c'}{\text{psi}}} + \frac{l_w \left(1.25 \sqrt{\dfrac{f_c'}{\text{psi}}} + 0.2 \dfrac{N_u}{\text{lbf}} \dfrac{\text{in}}{l_w} \dfrac{\text{in}}{h} \right)}{\dfrac{M_u}{V_u} - \dfrac{l_w}{2}} \right] \frac{h}{\text{in}} \frac{d}{\text{in}} \tag{11-32}$$

or

$$\frac{V_c}{\text{N}} = \left[0.05 \sqrt{\frac{f_c'}{\text{MPa}}} + \frac{l_w \left(0.1 \sqrt{\dfrac{f_c'}{\text{MPa}}} + 0.2 \dfrac{N_u}{\text{N}} \dfrac{\text{mm}}{l_w} \dfrac{\text{mm}}{h} \right)}{\dfrac{M_u}{V_u} - \dfrac{l_w}{2}} \right] \frac{h}{\text{mm}} \frac{d}{\text{mm}}$$

The steel contribution V_s is calculated by:

$$V_s = \frac{A_v f_y d}{s_2} \tag{11-33}$$

3

Current provisions for the seismic shear strength of walls are found in ACI 318-99 §21.6. The nominal shear strength V_n of structural walls is limited by:

$$\frac{V_n}{\text{lbf}} \leq \frac{A_{cv}}{\text{in}^2}\left(\alpha_c\sqrt{\frac{f_c'}{\text{psi}}} + \rho_n\frac{f_y}{\text{psi}}\right) \tag{21-7}$$

or $\quad \dfrac{V_n}{N} \leq \dfrac{A_{cv}}{\text{mm}^2}\left(\dfrac{\alpha_c}{12}\sqrt{\dfrac{f_c'}{\text{MPa}}} + \rho_n\dfrac{f_y}{\text{MPa}}\right)$

where $\quad \alpha_c = 3.0 \quad$ for $\quad \dfrac{h_w}{l_w} \leq 1.5$

$\quad \alpha_c = 2.0 \quad$ for $\quad \dfrac{h_w}{l_w} \geq 2.0$

and α_c varies linearly between 3.0 and 2.0 for $\dfrac{h_w}{l_w}$ between 1.5 and 2.0.

Furthermore, the shear strength of individual wall piers is limited by (§21.6.4.4):

$$\frac{V_n}{\text{lbf}} \leq 10\,\frac{A_{cp}}{\text{in}^2}\sqrt{\frac{f_c'}{\text{psi}}}$$

or $\quad \dfrac{V_n}{N} \leq 0.83\,\dfrac{A_{cp}}{\text{mm}^2}\sqrt{\dfrac{f_c'}{\text{MPa}}}$

3.2 Tests of HSC Shear Walls

Wallace (1998) reviewed 37 cyclic tests of shear walls performed in Japan as part of the "New RC" project to study HSC with high-strength steel reinforcement (Tables 6, 7 and Fig. 3). Concrete strength ranged from 55 MPa (7 900 psi) to 137 MPa (19 900 psi). Table 8 and Fig. 4 show that ACI 318-95 (or ACI 318-99) Eq. 21.7 provides conservative estimates of shear strength. Details of the tests can be found in Kabeyasawa and Hiraishi (1998) and in Gupta and Rangan (1996).

Take wall NO. 6 as an example (l_w includes the end piers):

$$\frac{h_w}{l_w} = \frac{2000\,\text{mm}}{1700\,\text{mm}} = 1.18, \quad \text{so} \quad \alpha_c = 3.0$$

4

$$V_n = A_{cv} \left(3 \sqrt{f_c'} + \rho_n \, f_y \right)$$

$$V_n = 66.93 \times 3.15 \left(3\sqrt{10745} + \frac{1.795}{78.74 \times 3.15} \times 205\,900 \right)$$

$$V_n = 379\,720 \quad \text{lbf} \; = 1689 \quad \text{kN}$$

However, the limit of individual walls governs:

$$V_n = 10 \, A_{cp} \sqrt{f_c'} = 10 \times 66.93 \times 3.15 \sqrt{10\,745} = 218\,540 \quad \text{lbf} \; = 972 \quad \text{kN}.$$

Therefore, the ratio of measured shear strength to ACI nominal shear strength prediction is

$$\frac{\text{Test}}{\text{ACI}} = \frac{1412 \text{ kN}}{972 \text{ kN}} = 1.45, \text{ which is the value reported in column 5 of Table 8 and is conservative.}$$

The ratio of measured shear strength to ACI nominal shear strength prediction for the 37 Japanese wall tests reported in column 5 of Table 8 has a mean of 1.49, a standard deviation of 0.31 and a coefficient of variation of 0.21.

Gupta and Rangan (1996) performed seven shear wall tests under static load. The walls have dimensions $h_w = l_w = 1000$ mm, $h = 75$ mm (Fig. 5), and a concrete strength of about 70 MPa. The horizontal shear reinforcement consists of ten bars on each face, spaced at 100 mm. For walls S1 to S6, the bars have cross-sectional area $A_v = 19.6$ mm^2, yield strength $f_y = 578$ MPa, and for wall S7, $A_v = 39.6$ mm^2, $f_y = 545$ MPa.

It turns out that the nominal shear strength based on Eqs. 11-31 and 11-33 controls for walls S1 and S4, and § 11.10.3 controls for the other walls (S2, S3, S5, S6 and S7). Table 9 shows that the mean of the ratio of experimental shear strength to nominal shear strength is 1.87. Thus, the ACI Code provisions are safe for these walls.

3.3 Recommendation

Since ACI 318-99 Chapter 21 falls outside the scope of work of the task force, no recommendation is made based on the Japanese tests. At this time, no change is recommended to ACI 318-99 Chapter 11 provisions concerning shear walls because the data are too limited (seven tests), the compressive strength of the walls tested is at the present limit of applicability of ACI 318-99 § 11.1.2, and the ACI predictions are safe.

4 NOTATION

a — shear span, distance between concentrated load and face of support

A_{cp} — area of concrete section, resisting shear, of an individual pier

A_{cv} — gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered

A_s — area of non-prestressed tension reinforcement

A_v — area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s

A_{vh} — area of shear reinforcement parallel to flexural tension reinforcement within a distance s_2

b_w — web width

d — effective depth of section

f_c' — specified compressive strength of concrete

f_y — specified yield strength of reinforcement

h — overall thickness of member

h_w — wall height

l_n — clear span measured from face to face of support

l_w — horizontal wall length

M_u — factored moment at section

N_u — factored axial load normal to cross section occurring simultaneously with V_u

s — spacing of shear reinforcement in direction parallel to longitudinal reinforcement

s_2 — spacing of shear reinforcement in direction perpendicular to longitudinal reinforcement, or spacing of horizontal reinforcement in wall.

V_c — nominal shear strength provided by concrete

V_n — nominal shear strength

V_s — nominal shear strength provided by steel reinforcement

V_u — factored shear force at section

ρ_n — ratio of area of distributed reinforcement parallel to the plane of A_{cv} to gross concrete area perpendicular to that reinforcement

ρ_w — $A_s / (b_w d)$

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Table 1: Comparison of Shear Strength of 68 high-strength (HSC),
32 medium-strength (MSC) and 133 low-strength concrete (LSC)
Deep Beams with Predictions by ACI Tan et al., 1998)

	V_n/V_{exp}		
	HSC	MSC	LSC
Mean	0.58	0.63	0.65
Standard Deviation	0.29	0.20	0.20
Coeff. of Variation	0.49	0.31	0.31

Table 2a Dimensions and Details for Beams B1.2, B2.0, B2.0B,
B2.0C, B3.0, and B3.0B (Foster and Gilbert, 1998. See Fig. 2a)

Beam	Dimensions in mm (1 in = 25.4 mm)			Web Reinforcement	
	L	A	D	V	H
B1.2	1450	850	1200	8 ×2 W 6 (W6@75)	5 × 2W6 (W6@175)
B2.0	1400	825	700	8 × 2 W 6 (W6@75)	3 × 2 W 6 (W6@135)
B2.0B	1400	825	700	Nil	Nil
B2.0C	1400	825	700	11 × 2W6 (W6@50)	Nil
B2.0D	1400	825	700	8 × 2 W 6 (W6@75)	Nil
B3.0	2100	1175	700	13 × 2W6 (W6@75)	3 × 2 W 6 (W 6@135)
B3.0B	2100	1175	700	Nil	Nil

Table 2b Dimensions and Details for Beams B2.0A and B3.0A
(Foster and Gilbert, 1998. See Fig. 2b)

Beam	Dimensions in mm (1 in = 25.4 mm)			Web Reinforcement	
	L	A	D	V	H
B2.0A	1400	675	700	19 × 2 W 6 (W6@75)	3 × 2 W 6 (W6@135)
B3.0A	2100	925	700	29 × 2W6 (W6@75)	3 × 2 W 6 (W6@135)

Table 3 Concrete material properties,
Australian HSC Deep Beam tests (Foster and Gilbert, 1998)

Beam	f_c' MPa	f_t MPa	ρ kg/m^3	E_0 GPa	Age, days
B1.2-1	91	6.23	2570	53.5	11
B1.2-2	96	6.41	2570	55.0	15
B1.2-3	80	6.30	2520	48.5	32
B1.2-4	80	6.31	2520	48.6	34
B2.0-1	83	5.94	2570	51.0	10
B2.0-2	120	7.13	2570	61.1	63
B2.0-3	78	6.22	2520	47.9	27
B2.0A-4	86	6.52	2520	50.2	66
B2.0B-5	89	6.30	2540	-	90
B2.0C-6	93	6.50	2580	-	92
B2.0D-7	104	7.20	2550	52.7	57
B3.0-1	80	5.84	2570	50.1	9
B3.0-2	120	7.15	2570	61.4	65
B3.0-3	77	6.20	2520	47.7	25
B3.0A-4	88	6.59	2520	50.7	75
B3.0B-5	89	6.30	2540	-	95

Table 4- Experimental failure loads and mode of failure
Australian HSC Deep Beam Tests (Foster and Gilbert, 1998)

Specimen	Failure load, kN	Failure mode
B3.0-1	1020	Tension failure
B3.0-2	1050	Tension failure
B3.0-3	1050	Tension failure
B3.0A-4	1550	Tension failure
B3.0B-4	870	Tension failure
B2.0-1	1590	Balanced failure
B2.0-2	1650	Tension failure
B2.0-3	1400	Tension failure
B2.0A-4	1900	Tension failure
B2.0B-5	1170	Tension failure
B2.0C-6	1460	Tension failure
B2.0D-7	1440	Tension failure
B1.2-1	2000	Crushing of loading column in compression
B1.2-2	2000	Crushing of loading column in compression
B1.2-3	2600	Balanced failure
B1.2-4	2100	Crushing of loading Column in compression

Table 5: Comparison of Shear Strength of Australian HSC Deep Beam Tests
with ACI 318-89 Predictions (Foster and Gilbert, 1998)

Specimen	Experimental Strength, kN	Predicted Strength, kN	V_n / V_{exp}
B3.0-1	1020	824	0.81
B3.0-2	1050	827	0.79
B3.0-3	1050	819	0.78
B3.0A-5	1550	1087	0.70
B3.0B-5	870	403	0.46
B2.0-1	1590	967	0.61
B2.0-2	1650	1162	0.70
B2.0-3	1400	937	0.67
B20.A-4	1900	984	0.52
B2.0A-5	1170	685	0.59
B2.0A-6	1460	934	0.64
B2.0D-7	1440	884	0.61
B1.2-3	2600	1787	0.69
Mean			0.66
Standard Deviation			0.103

Table 6 - List of Walls in Japanese New RC Project:
Horizontal Sectional Dimensions of all New RC Specimens
(Kabeyasawa and Hiraishi, 1998)

Specimens	Column		Wall panel	
	width b	depth d	thickness h	length l_w
NW*, W**, NO.*, M***	200 mm	200 mm	80 mm	1300 mm

Table 7 - List of walls in Japanese New RC Project: horizontal sectional dimensions of concrete, reinforcement and loading conditions (Kabeyasawa and Hiraishi, 1998)

Wall	f_c'	Axial load	Shear Span Ratio	Column main bar	f_y	Column spiral hoop	f_y	Column sub-tie	f_y	Wall web bar	f_y
	MPa	kN		mm	MPa	mm	MPa	mm	MPa	mm	MPa
NW-1	87.6	1764	2.00	12-D10	776	D6@40	1262	2-D6@40	100	2-D6@150	1001
NW-2	93.6	1764	1.33	12-D10	776	D6@40	1262	2-D6@40	1001	2-D6@150	1001
NW-3	55.5	1372	2.00	12-D10	840	5@40	1233	2-5@40	1233	1-D6@150	753
NW-4	54.6	1568	2.00	16-D10	840	5@40	1233	2-5@40	1233	1-D6@150	753
NW-5	60.3	1372	2.00	16-D10	840	5@40	1233	2-5@40	1233	2-D6@150	753
NW-6	65.2	1568	2.00	12-D13	776	5@40	1233	2-5@40	1233	2-D6@150	753
W-08	103.3	1764	0.66	12-D10	761	D6@40	1372	2-D6@40	1079	2-D6@150	1079
W-12	137.5	2313	0.66	12-D10	761	D6@40	1372	20D6@40	1079	2-D6@150	1079
NO.1	65.1	1568	1.33	16-D13	1009	D6@40	1295	2-D6@50	1395	2-D6@400	792
NO.2	70.8	1568	1.33	16-D13	1009	D6@40	1295	2-D6@50	1395	2-D6@230	792
NO.3	71.8	1568	1.33	16-D13	1009	D6@40	1395	2-D6@50	1395	2-D6@150	792
NO.4	103.4	2617	1.33	16-D13	1009	D6@40	1395	2-D6@50	1395	2-D6@0	792
NO.5	76.7	1568	2.00	16-D13	1009	D6@40	1395	2-D6@e0	1395	2-D6@e0	792
NO.6	74.1	1568	1.33	16-D13	1009	D6@40	1395	2-D6@50	1395	2-U6.4@122	1420
NO.7	71.5	1568	1.33	16-D13	1009	D6@40	1395	2-D6@50	1395	2-D6@80	792
NO.8	76.01	1568	1.33	16-D13	848	D6@40	1395	2-D6@50	1395	2-D6@55	792
M35X	62.6	1764	2.00	12-D10	848	D6@40	810	2-D6@60	810	2-D6@100	810
M35H	68.6	1921	2.00	12-D10	848	D6@40	810	2-D6@60	810	2-D6@100	810
P35H	66.5	1862	2.00	12-D10	848	D6@40	810	--	--	2-D6@100	810
M30H	61.4	1470	2.00	12-D10	848	D6@40	810	2-D6@60	810	2-D6@100	810
MW35H	59.7	1666	2.00	12-D10	848	D6@40	810	2-D6@60	810	2-D6@100	810

Table 8 - HSC Wall Tests Conducted in Japan (Wallace 1998)

Specimen ID	V_{max} kN	$\dfrac{V_{max}}{V_u @ M_n}$	$\dfrac{V_n Eq.21.7}{V_u @ M_n}$	$\dfrac{V_{max}}{V_n Eq.21.7}$	$\dfrac{V_{max}}{V_n Eq.10}$	$\dfrac{V_{max}/MPa}{\sqrt{f_c'/MPa}}$	$\dfrac{\rho_n f_y}{f_c'}$	$\dfrac{A_{vf} f_y}{A_{cv} f_c'}$	$\dfrac{\rho_t f_{yt}}{f_c'}$
NW-1 (1)	1062	1.03	0.91	1.13	1.67	0.83	0.061	0.16	0.23
NW-2 (2)	1468	0.95	0.61	1.55	2.23	1.12	0.057	0.15	0.21
NW-3 (3)	717	0.84	0.52	1.62	1.42	0.71	0.036	0.22	0.24
NW-4 (4)	784	0.75	0.42	1.78	1.48	0.78	0.037	0.29	0.25
NW-5 (5)	900	0.86	0.69	1.25	1.54	0.85	0.067	0.29	0.22
NW-6 (6)	1056	0.86	0.59	1.45	1.52	0.96	0.062	0.31	0.21
W-08 (7)	1670	0.35	0.24	1.48	2.42	1.21	0.056	0.14	0.24
W-12 (8)	1719	0.32	0.22	1.46	2.16	1.08	0.042	0.10	0.18
NO. 1 (9)	1101	0.30	0.13	2.25	1.21	1.00	0.024	0.48	0.31
NO. 2 (10)	1255	0.33	0.17	1.90	1.32	1.10	0.039	0.46	0.28
NO. 3 (11)	1379	0.35	0.22	1.60	1.44	1.20	0.059	0.47	0.31
NO. 4 (12)	1697	0.37	0.20	1.84	1.50	1.23	0.041	0.32	0.22
NO. 5 (13)	1159	0.45	0.30	1.50	1.17	0.97	0.055	0.44	0.29
NO. 6 (14)	1412	0.33	0.23	1.45	1.45	1.21	0.126	0.50	0.30
NO. 7 (15)	1499	0.36	0.23	1.57	1.57	1.30	0.111	0.51	0.31
NO. 8 (16)	1639	0.37	0.23	1.66	1.66	1.38	0.151	0.51	0.29
M35X (17)	1049	0.63	0.54	1.17	1.95	0.98	0.104	0.25	0.15
M35H (18)	1055	0.61	0.55	1.12	1.87	0.94	0.094	0.23	0.14
P35H (19)	959	0.57	0.55	1.04	1.73	0.98	0.097	0.24	0.07
M30H (20)	1020	0.67	0.58	1.15	1.91	0.96	0.106	0.25	0.16
MW35H (21)	1012	0.63	0.54	1.16	1.91	0.96	0.109	0.26	0.16
MAE03 (22)	1460	0.27	0.19	1.46	2.19	1.10	0.066	0.25	
MAE07 (23)	1676	0.28	0.18	1.51	2.31	1.26	0.110	0.29	
W48M6 (24)	1516	0.57	0.52	1.10	1.62	0.81	0.054	0.10	
W48M4 (25)	1479	0.61	0.55	1.12	1.58	0.79	0.050	0.09	
W72M8 (26)	2066	0.57	0.43	1.33	2.21	1.10	0.088	0.16	
W72M6 (27)	2015	0.59	0.46	1.29	2.15	1.08	0.081	0.15	
W72M8 (28)	2128	0.59	0.48	1.23	2.04	1.02	0.071	0.13	
W96M8 (29)	2483	0.55	0.38	1.43	2.39	1.91	0.092	0.17	
SMZ01 (30)	1154	0.37	0.29	1.30	1.25	0.62	0.025	0.09	0.02
SMZ03 (31)	2081	0.33	0.14	2.35	2.25	1.13	0.025	0.18	0.02
W8N18 (32)	882	0.78	0.71	1.85	1.85	0.92	0.137	0.18	0.15
W8N13 (33)	762	0.80	0.87	1.53	1.53	0.77	0.126	0.17	0.14
W8N8H (34)	689	0.76	0.92	1.38	1.38	0.69	0.126	0.20	0.13
TAK01 (35)	971	0.59	0.62	1.52	1.52	0.76	0.077	0.17	0.10
TAK02 (36)	987	0.55	0.59	1.55	1.55	0.77	0.111	0.20	0.10
TAK03 (37)	1288	0.52	0.43	2.02	2.02	1.01	0.077	0.17	0.10

$$V_n = \text{the larger of} \left\{ A_{cv} 6\sqrt{\frac{f_c'}{psi}} ; \frac{A_v f_y}{4} \right\} \le A_{cv} 10\sqrt{\frac{f_c'}{psi}} \qquad (10a)$$

$$V_n = \text{the larger of} \left\{ A_{cv} \frac{1}{2}\sqrt{\frac{f_c'}{MPa}} ; \frac{A_v f_y}{4} \right\} \le A_{cv} 0.83\sqrt{\frac{f_c'}{MPa}} \qquad (10b)$$

Table 9 Australian Shear Wall Tests (Gupta and Rangan, 1996)

Walls	S1	S2	S3	S4	S5	S6	S7
f'_c MPa	79.3	65.1	69.0	75.2	73.1	70.5	71.2
f'_c psi	11500	9 440	10 000	10 900	10 600	10 220	10 320
Axial kN	0	610	1 230	0	610	1 230	610
Axial kips	0	137.1	276.5	0	137.1	276.5	137.1
$10\sqrt{f'_c}\,h\,d$	99.651	90.286	92.925	97.017	95.672	93.942	94.600
(Eq. 11.31) V_c	32.885	57.214	85.965	32.015	58.992	86.300	58.572
(Eq. 11.33) V_s	40.712	40.712	40.712	40.712	40.712	40.712	77.555
V_c+V_s kips	73.597			72.73			
Fail kN	427.8	719.6	850.7	600.0	790.2	970.0	800.0
Fail kips	96.18	161.78	191.25	134.89	177.65	218.08	179.86
Test / ACI	1.31	1.79	2.06	1.85	1.86	2.32	1.91

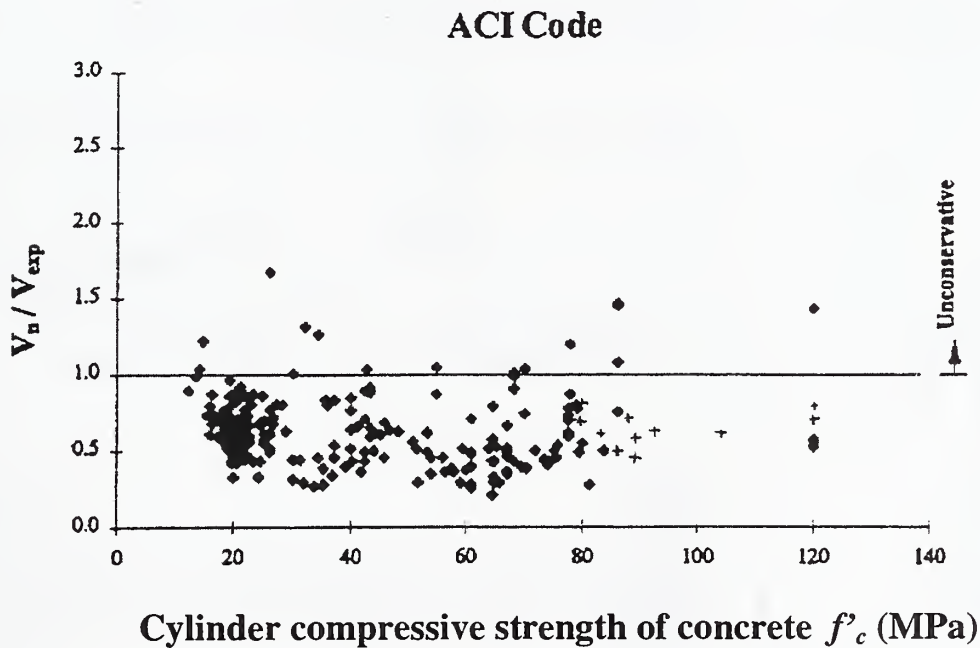Test / ACI: mean = 1.87   standard deviation = 0.31   CoV=0.16

Fig 1 Shear Strength of Deep Beams
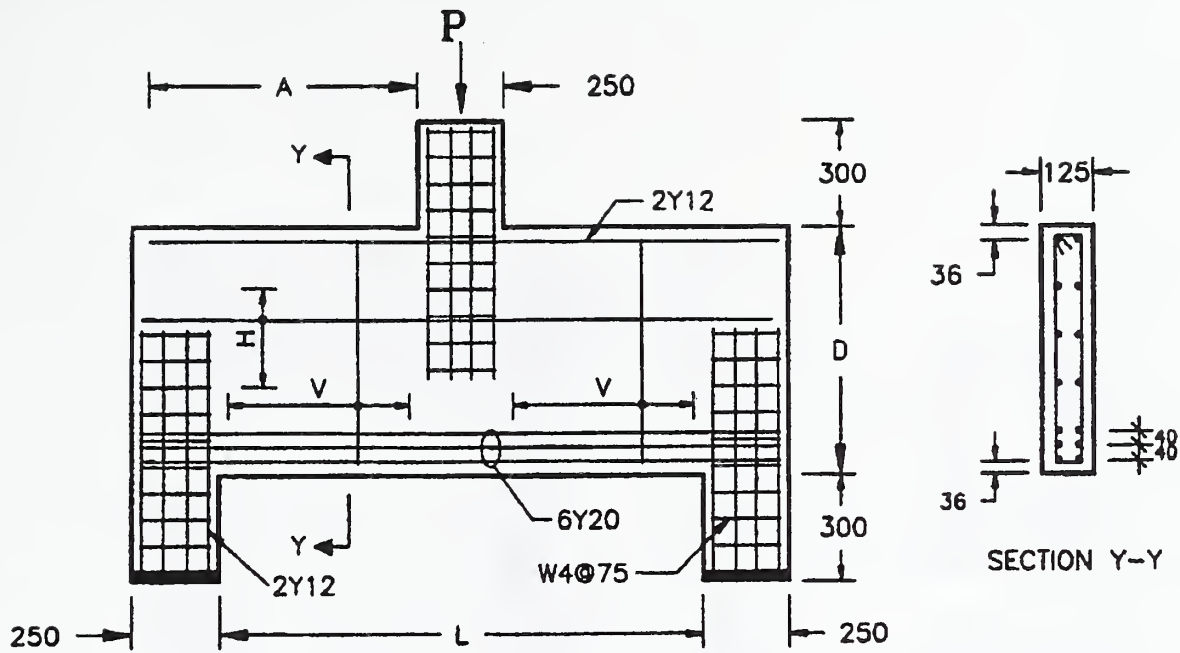(Tan et al., 1998, Foster and Gilbert, 1998)

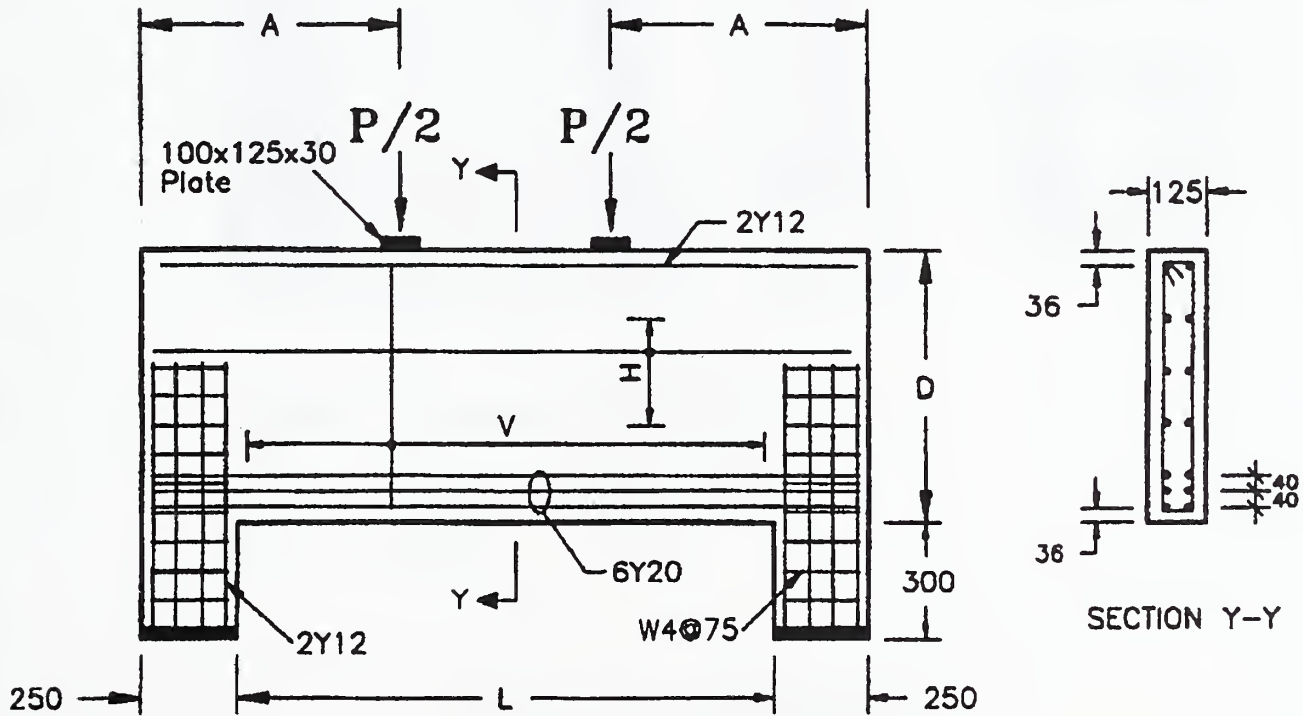Fig 2a – Deep Beam Test (Foster and Gilbert, 1998). See Table 2a

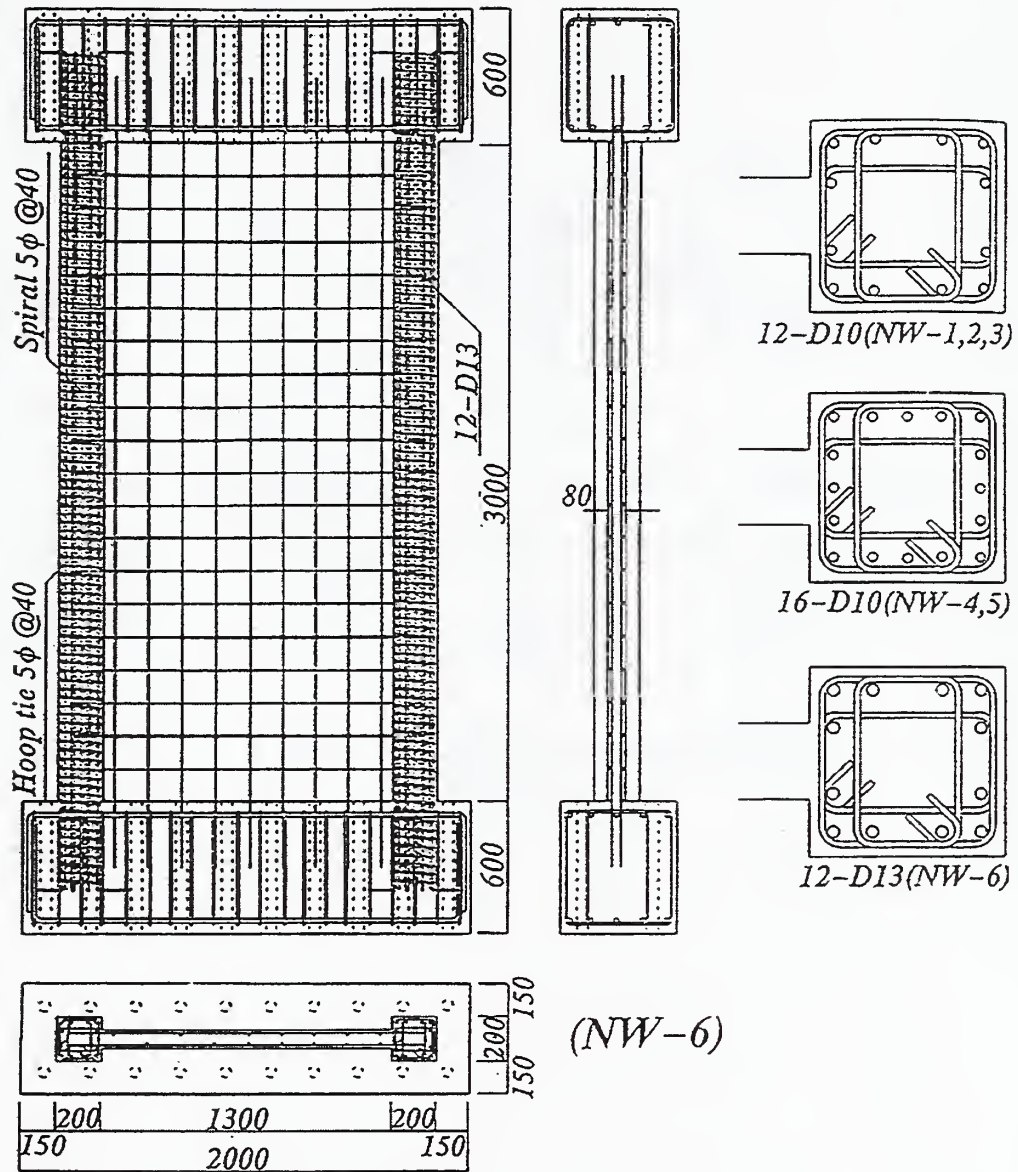Fig 2b –Deep Beam Test (Foster and Gilbert (1998). See Table 2b

Spiral 5φ @40

Hoop tie 5φ @40

12–D13

600

3000

600

80

12–D10(NW–1,2,3)

16–D10(NW–4,5)

12–D13(NW–6)

(NW–6)

150

200

150

200

1300

200

150

2000

150

Fig 3 – "New RC" Wall Tests (Japan)
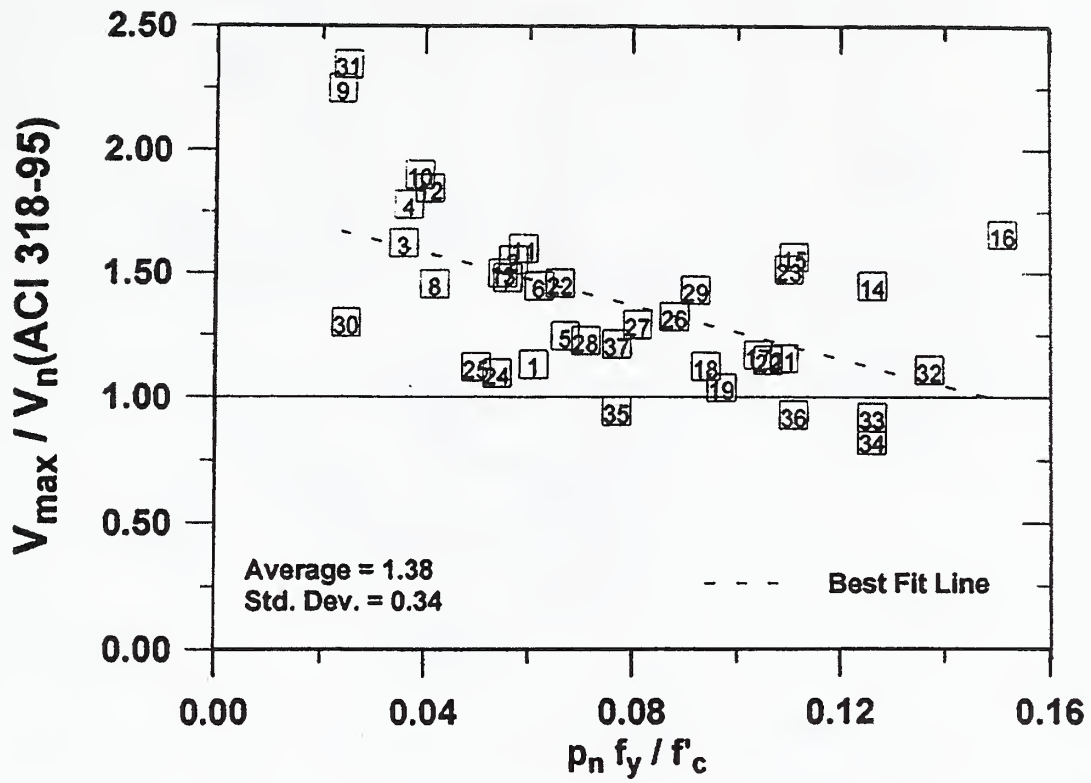(Kabeyasawa and Hiraishi, 1998)
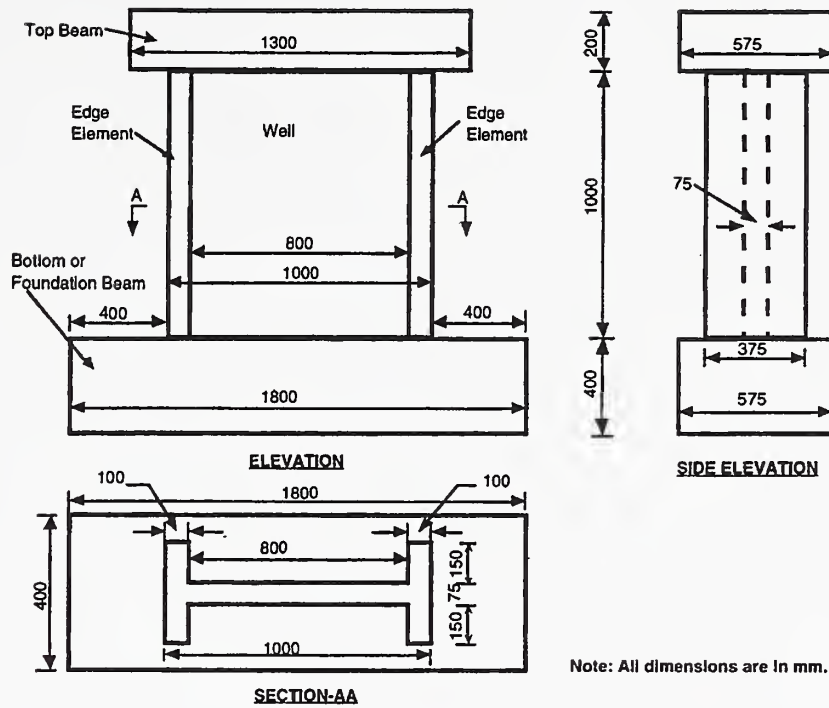
Fig 4 – Wall shear strength: ACI 318-95
(Wallace, 1998)

Figure 5 Dimensions of Test Wall (Gupta and Rangan, 1996)

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